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PRELIMINARY DESIGN OF A SUPPRESSIVE STRUCTURE FOR A MELT-LOADING OPERATION

by
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The report discusses the preliminary design and analyses of a suppressive structure for a melt loading operation having a capacity of 2500 lb of high explosive. The basic structural concept consists of a complete box-shaped steel frame supporting vented panels. The structure is designed to withstand blast loads and fragment impacts.

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PREFACE

The work described in this report was authorized under PA, A 5751264, Preliminary Design of a Suppressive Structure for a Melt-Loading Operation. It was performed from Jan 1974 to Sept 1974.

SUMMARY

This paper discusses the preliminary design and analysis of a suppressive structure for a melt-loading operation having a capacity of 2500 lb of high explosive. The primary purpose of a suppressive structure is to reduce the required spacing of facilities which contain and process high-explosives by reducing the overpressure outside of the structure and controlling the fragments from an internal explosion.

A development program is being supported by the Edgewood Arsenal to establish the technology for designing suppressive structures so that they can be routinely applied to explosive processing operations. This program is divided into several phases which will culminate in a full-scale design and test.

Design loads for the structure include those due to the internal blast wave plus those due to the long term pressure buildup in the structure generated by the heat of explosion. Appropriate methods of analyses were used to assess the effect of both the dynamic and quasi-static loads in the structural design. The design was also influenced by the fragmentation requirements which often controlled structural sizing rather than blast loading.

The suppressive structure consists of a structural steel frame to which vented steel panels are attached. Several panel concepts were developed and designed for the full scale structure. These panels will be tested in full scale against simulated primary fragments. Blast attenuation tests will also be conducted using subscale panels to determine venting characteristics. In addition, a one-quarter scale replica model structure will be tested to obtain additional information on venting and on the structural integrity of the frame and panels. One concept for a quarter-scale frame design is presented in this report. Venting and structural response data obtained in tests of a quarter-scale structure will be used to design a full-scale prototype. Its venting characteristics, fragment suppression capabilities, and structural integrity will then be verified by full-scale testing.

This report is a reprint of a paper presented at the 16th Explosive Safety Seminar, Hollywood Beach, Florida, September 1974.

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I. INTRODUCTION

Hazards produced by accidental explosions within facilities that contain and process high explosives have concerned safety engineers for many years. One obvious way to reduce the hazards is to separate such facilities as far apart as possible to avoid the potential for propagation of such an explosion and also to place the facilities as far away from populated areas or other nonrelated operations as possible. Another approach, which is the subject of this report is to use a suppressive structure to contain the fragments and suppress the air blast wave from the detonation in order to reduce the required spacing to a minimum. This report represents a progress report on the design and analysis of a structure to contain the fragments and suppress the air blast from the detonation of 2500 lb of Composition-B explosive being processed in a melt kettle.

The technology for vented suppressive structures has not yet reached the stage where the design is a straightforward process. Most of the development to date has taken place at the NASA National Space Technology Laboratories (NSTL) and has been for the containment of relatively small explosive charges.[1,2,3]*

To establish the technology for the design of suppressive structures, development programs are being conducted to determine acceptable panel configurations and frame designs to defeat the fragment hazard and to reduce the air blast to acceptable levels. The present program, supported by the Edgewood Arsenal, will develop the technology for suppressive structures so that they can be routinely applied to explosive processing operations.

This particular development program is for the Category 1 shield and represents the probable upper limit in charge weight for which suppressive structures will be designed. The program is divided into several phases which will culminate in a full-scale design and test. The phases are:

Fragment Hazard Evaluation

This evaluation will consist of two subphases: (1) a melt kettle fragment analysis whereby the worst case fragment from a melt kettle is determined by actual tests, and (2) panel penetration tests whereby simulated full-scale primary fragments are fired at full scale panel components.

*Superscript numbers denote references included at the end of the paper.

Blast Hazards Analysis

In the blast hazards analysis the venting characteristics of the panels will be investigated in order to determine qualitative comparisons between candidate panels. In addition, a one-quarter scale replica of a full-scale prototype structure will be designed and tested. This test will provide information on the venting characteristics of the panels and also on the structural integrity of the frame and panels.

Full-Scale Prototype Design and Test

Information from the fragment hazard evaluation and blast hazards analysis, including the structural information from the quarter-scale tests, will be used to design the full-scale prototype. Its venting characteristics, fragment suppression capabilities and structural integrity will then be verified by testing.

As related to the development program, this paper describes the panel concepts which will be evaluated in the fragment hazard evaluation and blast hazard analysis, as well as the frame which will be evaluated in the quarter scale testing.

II. DESIGN REQUIREMENTS

Primary requirements for a suppressive structure designed for a melt-loading operation having a capacity of 2500 lb of high explosives are:

(1) Maintain structural integrity for a charge weight of 3125 lb, thus ensuring a margin of safety of 25% based on charge weight.

(2) Contain all primary fragments within the structure. Reference 4 defines this requirement as a 1-lb fragment with a velocity of 7200 fps.

(3) Reduce the side-on overpressure outside the structure to 5 psi at a distance of 75 ft from the center of a 2500-lb high explosive charge.

(4) Floor area of approximately 40 ft by 40 ft with an internal volume of about 64,000 ft^3.

(5) Incorporate to the extent possible existing technology and test results of past suppressive structure designs.

III. DESIGN APPROACH

Many concepts were considered in the design of this suppressive structure. The most obvious design configuration for a structure which is to contain internal pressure is either spherical or cylindrical. Although such configurations result in much more efficient structures for resisting the internal blast loading, it so happens that the steel thickness in the walls of the structure required to defeat the primary fragment are much greater than the thickness required to resist the blast loading. In addition, venting the walls of curved cylinders and spheres significantly complicates construction over that for a straight-sided rectangular box. After considering the alternatives, the following guidelines were established to define the design approach for this suppressive structure.

- Structure would be square or rectangular in shape
- All-steel construction would be used for both panels and frame. Also, frame members which form the sides and the roof of the structure would continue through the floor to complete the rectangular box.
- Maximum venting would be designed into the walls and roof of the structure consistent with the external overpressure requirements.
- The structure would be designed to have a central charge location which would result in an intermediate vented steel grid floor.
- In order to reduce structural weight to a minimum, extensive plastic deformation of both panels and frame is allowed.

Following these guidelines, several panel concepts were developed and a one-quarter scale replica of the prototype frame developed for the quarter-scale testing.

IV. PANEL CONCEPTS

Four different panel designs were formulated using standard structural angles, perforated plates and tubes. All four panels were designed for the full-scale prototype structure. Since quarter-scale tests will first be conducted, the panels tested will be replica models of those presented here. For this type of model, pressure, stress and strain levels are the same as in the prototype; length and time become 1/4 the value; and the charge weight required to produce these analogous conditions is $(1/4)^3$ times the prototype charge weight. Furthermore, any nondimensional parameters, such as the vented area ratio, remain the same in the model as in the prototype.

In designing the panels, an effective venting area ratio was first computed to provide the required side-on overpressure reduction. Using the equation[5]

$$\alpha_{eff} = \frac{P'_s R^3 X}{(976.3)^2 W^{4/3}} \tag{1}$$

where

$P'_s = 5$ psig $\qquad X = 40$ ft
$R = 75$ ft $\qquad W = 2500$ lb

the effective venting ratio, α_{eff}, needed is 0.013. From this, the venting ratio, α_i, of each element in a panel is computed by assuming that[5]

$$\frac{1}{\alpha_{eff}} = \sum_{i=1}^{N} \frac{1}{\alpha_i}, \quad N = \text{number of elements} \tag{2}$$

In each element, α is defined as the ratio of open area divided by the total area of the wall.

To defeat the primary fragment threat using four spaced plates requires a total thickness of 2.35 in. of steel.[4,6] All the panels were therefore designed such that each configuration would provide at least four spaced plates with the necessary total steel thickness. Each geometry was then checked to ensure that the dynamic and quasi-static blast loads were also contained. Since the structure is to be designed to contain the detonation of 3125 lb of explosive, the dynamic design loads, the reflected impulse and pressure, were determined from data by Goodman[7] to be 2.43 psi-sec and 4057 psi, respectively. The quasi-static design load was determined by computing the charge weight to structure volume ratio and using the methods presented by Baker and Westine[5]. Since the effective venting of the structure is very small, the quasi-static pressure rise of 165 psig for an unvented enclosure is used.

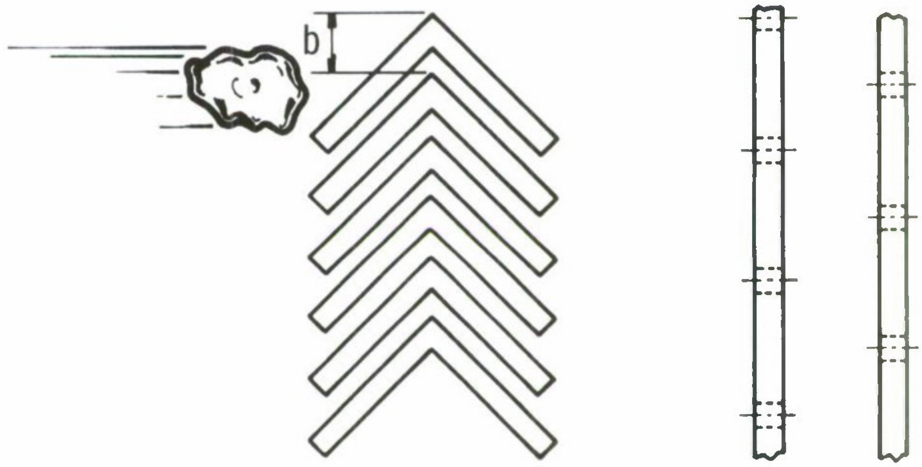
The first panel design (Figure 1) uses a set of angles spaced so that the specific impulse can be assumed to be absorbed and dispersed by the angles and the perforated plates need only be sufficient to withstand the quasi-static pressure. Since the angles are spaced to provide about a 30% vent area, the quasi-static pressure does not load them.

In this case, the angles provide the majority of the material thickness required to stop the primary fragments. Using this criterion, several angles were selected and checked against the dynamic loading using the following equation:[9]

$$\frac{i_r^2 b^2 L}{\rho A M_p} = 16 \frac{w_o}{L}$$

where

$i_r = 2.43$ psi-sec $\qquad w_o = 0.15\,L$ (in)
b – angle spacing (in) $\qquad A$ – cross-section area (in^2)
$L = 116$ in $\qquad \rho = 0.283/386$ lb-sec^2/in^4



	Full Scale	1/4 - Scale
Angles	L 3-$^1/_2$ x 3-$^1/_2$ x $^1/_2$	L $^7/_8$ x $^7/_8$ x $^1/_8$
Angle Spacing	b = 1.24 in	b = 0.31
Plates	$^9/_{16}$ in	$^9/_{64} \simeq$ 10 Ga
Holes	$^1/_2$ in Diameter	$^1/_8$ in Diameter
Hole Spacing	2.7 in	0.675 in

FIGURE 1. PANEL CONCEPT 1

From these results, 3-1/2 X 3-1/2 X 1/2 angles were selected.

Two perforated plates were sized to back the angles and contain the quasi-static pressure by allowing the plates to develop uniaxial membrane action as well as bending. From[8]

$$\frac{P_{\max}\, x^2}{\sigma_y h^2} = \frac{3}{4} + \frac{4}{5}\frac{w_o}{h} \tag{4}$$

where

$P_{\max}$ = 165 psi	L = 116 in
X = 58 in	$w_o = 0.15\,L$
σ_y = 36,000 psi	

the thickness of each plate was computed to be 9/16 in. allowing for the perforations.

The second panel design (Figure 2) is similar to the first. However, in this case, the size and spacing of the angles were chosen such that the required material is distributed between the angles and plates. Furthermore, the vent area of the angles is larger than on the first configuration so that it can be assumed that the plates will also be loaded dynamically. After some selective computations using Eq. (2), 5 X 5 X 3/4 angles were chosen for this panel.

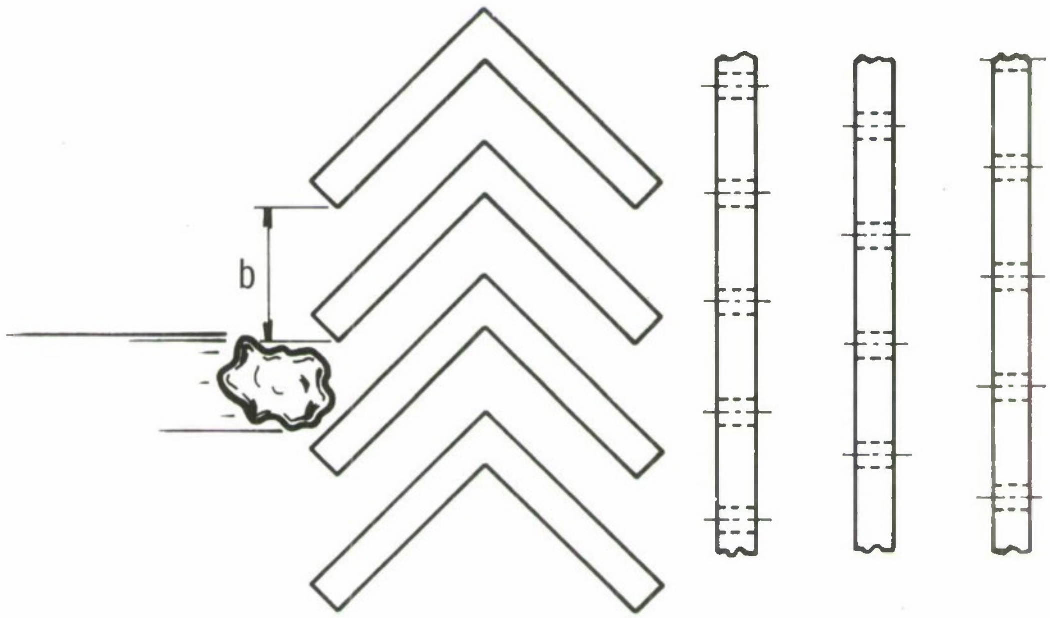
In sizing the plates, both the dynamic and quasi-static loadings were considered. For the dynamic loading, the following equation was used to determine the total plate thickness allowing uniaxial membrane and bending action[8]

$$\frac{i_r{}^2 X^2}{\rho \sigma_y h^4} = \frac{2}{3}\left(\frac{w_o}{h}\right) + \frac{32}{35}\left(\frac{w_o}{h}\right)^2 \tag{5}$$

Arbitrarily selecting three plates to back the angles, a plate thickness of 5/8 in. was computed. Since the total plate thickness required using Eq. (4) is greater than that for Eq. (3) (quasi-static), the impulsive load governs in this case.

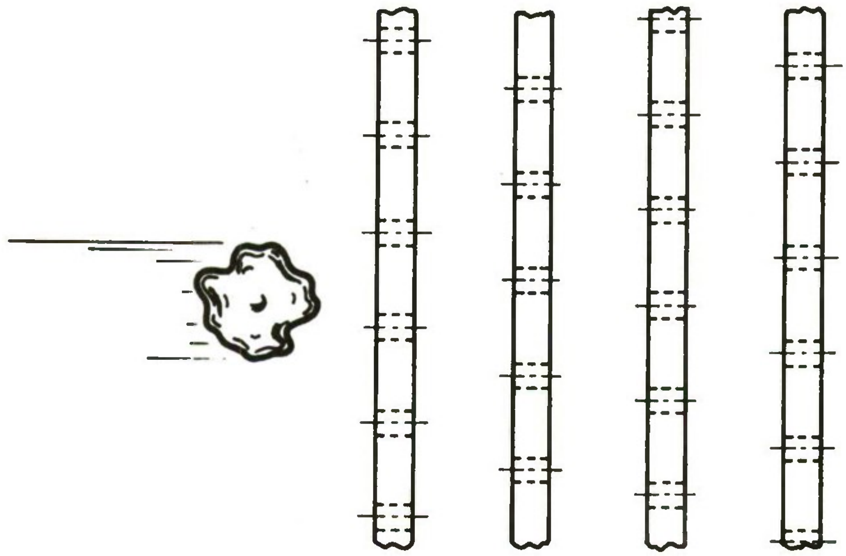
Having sized the plates, a check was made to determine if sufficient material was available to defeat the primary fragment threat. Since the angles provide 1 in. of steel and the plates 1-7/8 in., the fragment criteria have been met.

For the third panel design (Figure 3), a set of four perforated plates are used to contain the primary fragments, as well as to carry the total air blast loading. Since a total thickness of 2.35 in. of steel is requireu to meet the primary fragment threat, each plate should be one-fourth the total thickness plus the thickness required to account for the vented area. Each plate was computed to be 11/16 in. thick. From Eqs. (3) and (4), the plate thicknesses required to carry the quasi-static and dynamic loads are less than those needed against the



	Full Scale	1/4 - Scale
Angles	L 5 x 5 x 3/4	L 1-1/4 x 1-1/4 x 3/16
Angle Spacing	b = 2.71 in	b = 0.678
Plates	5/8 in	5/32 in $\simeq$ 3/16 in
Holes	1/2 in Diameter	1/8 in Diameter
Hole Spacing	2.19 in	0.548 in

FIGURE 2. PANEL CONCEPT 2



	Full Scale	1/4 - Scale
Plates	11/16 in	$11/64 \simeq 3/16$ in
Holes	3/4 in Diameter	3/16 in Diameter
Hole Spacing	2.90 in	0.725 in

FIGURE 3. PANEL CONCEPT 3

primary fragments. Therefore, the four perforated plates will be more than adequate against the blast loading.

The last panel design (Figure 4) we considered consists of tubes in a staggered arrangement. The tubes were sized and spaced such that a primary fragment would encounter at least 2.5 in. of steel with four effective layers as it traverses the panel. The spacing must also provide the effective venting area ratio required.

In sizing the tubes, it became evident that this panel concept would be the heaviest of the four as well as require considerably more fabrication time. Consequently, this design has been dropped from the testing program.

V. PRELIMINARY DESIGN OF QUARTER-SCALE FRAME

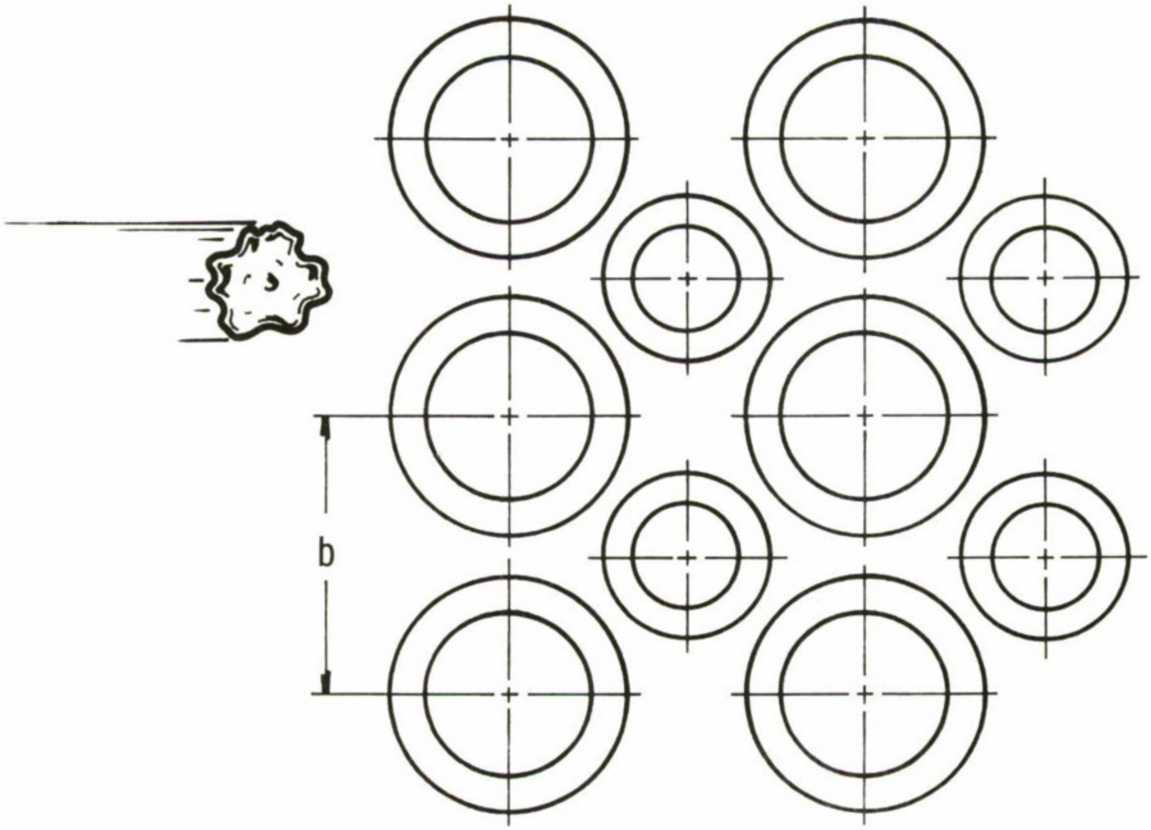
This section discusses the design of the frame for the quarter-scale model which is to be tested in the Blast Hazards Evaluation Phase of the development program. The frame was designed in quarter scale for the loads which the quarter-scale model will experience. It could just as easily have been designed in full scale and geometrically reduced to model size except that, in quarter scale, stock structural members can be more readily selected to aid in fabrication. Most, if not all, full-scale members must be built-up members.

According to Hopkinson's scaling, for scaled charge weights (3125 lb full scale and 50 lb quarter scale) quasi-static and dynamic pressures on the inside of the model will be identical to those of the full-scale structure. However, as pointed out earlier, time scales by the ratio of geometric lengths and is thus reduced by a factor of four in quarter scale. Hence, pressure loads on the design of the quarter-scale frame will be the same as those used for the design of the panels (designed in full scale), and the impulse will be reduced to one-fourth of the full-scale value. Summarizing, the loads are:

- Maximum quasi-static pressure: P_{qs} = 165 psi
- Maximum reflected pressure from the blast wave: P_r = 4057 psi
- Maximum reflected specific impulse: i_r = 0.6 psi-sec

In the design of the frame, the following assumptions and guidelines were adhered to:

- Although attenuation of the load on the frame due to the panel response is possible for the dynamic loading, such effects were neglected. For the quasi-static loading, which has a very long duration relative to the response time of the panels or frame, such attenuation is not probable.
- The maximum allowable deflection of the side members was set as 0.15 L (15% of the member's length).



	Full Scale	$^1/_4$ - Scale
Large Tubes	5 in OD, 3.5 in ID	1.25 in OD, 0.875 in ID
Small Tubes	3.5 in OD, 2.25 ID	0.875 in OD, 0.56 in ID
Spacing	b = 5.6 in	b = 1.4 in

FIGURE 4. PANEL CONCEPT 4

- A maximum strain in the material of 10% was allowed for dynamic loading.

- A maximum allowable tensile or compressive stress of 45,000 psi was set for the quasi-static loading. This is approximately midway between the minimum yield and minimum ultimate tensile properties of the material and will probably result in some deformation of the structure and strain hardening in order to develop this strength.

- Strain rate effects on the material strength were ignored since quasi-static loads governed the design.

The configuration developed for the frames is shown in Figure 5. Intersecting box beams form the roof and floor of the structure, and vertical members with no cross supports form the four sides. Box beams were chosen for the main structural elements in the frame for several reasons:

(1) The members are compact, which is essential for both dynamic and plastic deformations.

(2) The experience with members of similar proportions in Reference 2 showed good resistance to local buckling under high plastic deformations.

(3) The section has a high shear strength to bending strength ratio.

(4) Good support is provided for the panels by box beams because the panel loads are transferred directly through the webs of the box rather than through flanges as would be the case for wide-flange beams.

Preliminary sizing of frame members was made using equations developed by Westine and Baker[9] or derived following the same procedures but using different deformed mode shape assumptions. A more detailed analysis of the elastic-plastic deformations in the frame will also be made using a finite element computer program. The equations used for preliminary sizing of the frame are summarized below:

Clamped Beam

Deformed shape and strain equation:

$$w = \frac{16w_o}{L^4}\left[x^2 - \left(\frac{L}{2}\right)^2\right]^2 \qquad (6)$$

$$\epsilon_{max} = \frac{32c}{L}\left(\frac{w_o}{L}\right) \qquad \text{at ends} \qquad (7)$$

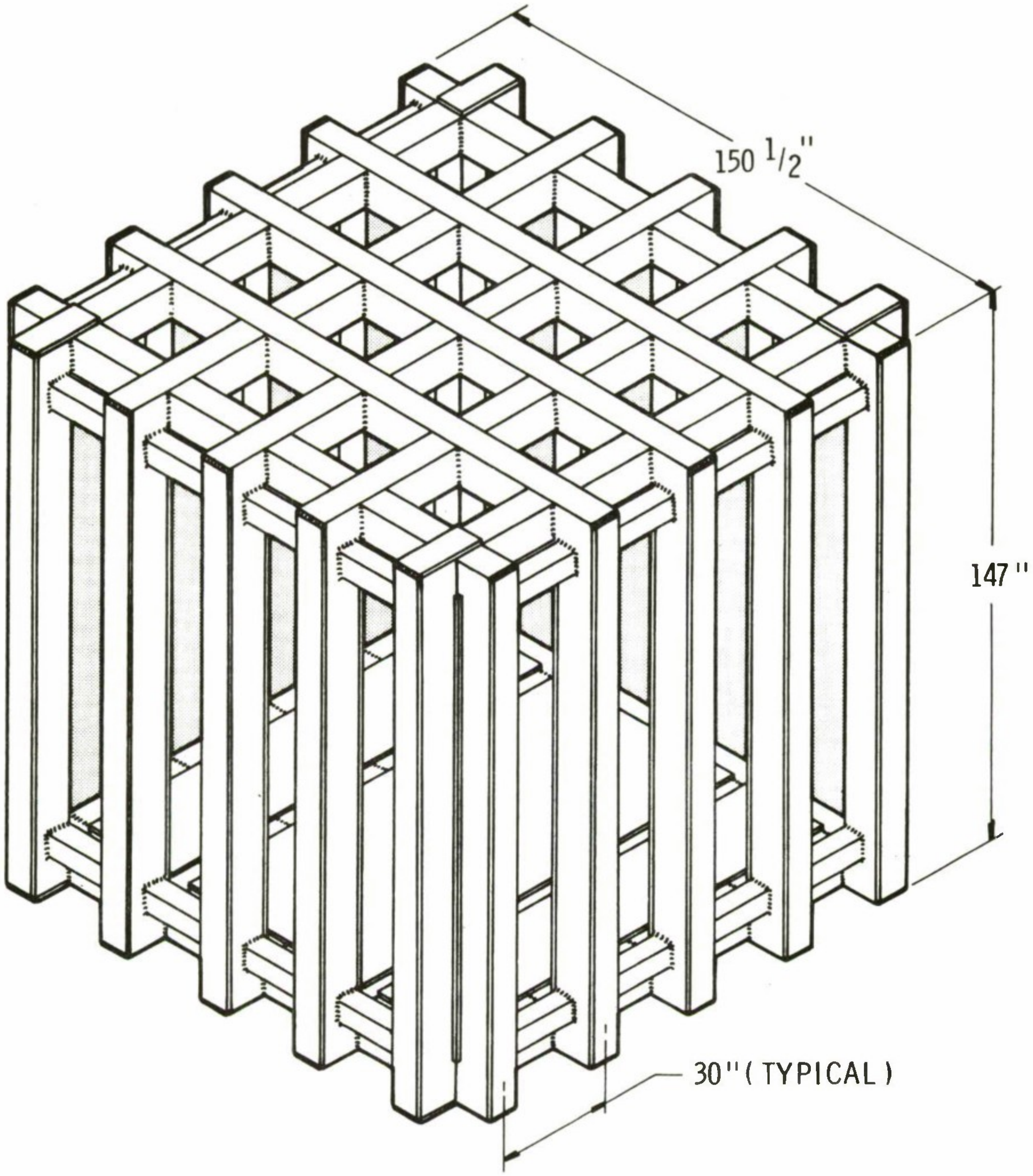


FIGURE 5. QUARTER-SCALE FRAME

Elastic behavior–impulsive load:

$$M_{max} = 2.236\, i_r b \sqrt{\frac{EI}{\rho A}} \qquad \text{at ends} \qquad (8)$$

$$V_{max} = \frac{13.417 i_r b}{L} \sqrt{\frac{EI}{\rho A}} \qquad \text{at ends} \qquad (9)$$

Plastic behavior–impulsive load:

$$\frac{i_r^2 b^2 L}{M_p \rho A_{eff}} = 24.64 \left(\frac{w_o}{L}\right) \qquad (10)$$

Plastic behavior–quasi-static load:

$$\frac{P_{qs} b L^2}{M_p} = 23.09 \qquad (11)$$

Simply-Supported Beam

Deformed shape and strain equation:

$$w = w_o \cos\left(\frac{\pi x}{L}\right) \qquad (12)$$

$$\epsilon_{max} = \frac{\pi^2 c}{L}\left(\frac{w_o}{L}\right) \qquad \text{at center} \qquad (13)$$

Elastic behavior–impulse load:

$$M_{max} = i_r b \sqrt{\frac{2EI}{\rho A}} \qquad (14)$$

$$V_{max} = \frac{\pi i_r b}{L} \sqrt{\frac{8EI}{\rho A}} \qquad (15)$$

Plastic behavior–impulsive load:

$$\frac{i_r^2 b^2 L}{M_p \rho A_{eff}} = 12.566 \left(\frac{w_o}{L}\right) \qquad (16)$$

Plastic behavior–quasi-static load:

$$\frac{P_{qs} b L^2}{M_p} = 9.87 \tag{17}$$

where:

c	–distance to external fibers
w	–lateral deflection in the beam at any point x
w_o	–maximum center deflection
L	–beam length
ϵ_{max}	–maximum strain
M_{max}	–maximum moment under impulsive loading for elastic behavior
V_{max}	–maximum shear under impulsive loading for elastic behavior
M_p	–fully plastic moment in the beam
A	–beam cross-sectional area
EI	–beam bending rigidity
b	–loaded width (approximately equal to the panel width)
ρ	–material density
A_{eff}	–area of beam plus an effective area of the panel to account for both panel and beam masses for dynamic response calculations.

Loading terms of pressure and impulse have been defined previously. To apply the equations for impulsive loading, w_o/L is computed from Eqs. (7) or (13) for a maximum strain of 10% (w_o/L is limited to 0.15, maximum). The required plastic moment, M_p, and effective area, A_{eff}, are then determined from Eqs. (10) or (16). For quasi-static loading M_p is determined from the force balance in Eqs. (11) or (17).

Because the members in the frame are neither fully clamped nor simply supported, calculations were performed for both conditions, and judgment was applied to select a member which fit somewhere inbetween the two extremes. Requirements for the frame members can be determined from Eqs. (10) and (11) for clamped beams and from Eqs.

(16) and (17) for simply supported beams. These requirements are expressed in terms of the plastic moment for the quasi-static loading, and in terms of the product of the plastic moment and the cross-sectional area for the dynamic loading. In addition, the quasi-static shearing load at the ends of the members is simply the product of the quasi-static pressure and the total panel area which the member must support. Using the geometry of Figure 5, these requirements for the vertical members in the sides of the structure are:

Clamped Beam

Quasi-static load:	$M_p = 3.91 \times 10^6$ lb-in. $V = 297{,}000$ lb
Impulsive loading:	$M_pA = 11.4 \times 10^6$ lb-in.^{3}

Simply-Supported Beam

Quasi-static loading:	$M_p = 9.14 \times 10^6$ lb-in. $V = 297{,}000$ lb
Impulsive loading:	$M_pA = 22.4 \times 10^6$ lb-in.^{3}

An 8 X 10-in. box beam with a 1/2-in. wall and a 1/2-in. reinforcing plate across each 8-in. dimension (see Figure 6) was chosen as the best structural component to satisfy the above requirements. For a maximum allowable tensile stress of 45,000 psi and a maximum shearing stress of 30,000 psi,* the beam develops the following properties:

$$M_p = 4.51 \times 10^6 \text{ lb-in.}$$

$$V = 330{,}000 \text{ lb}$$

$$M_pA = 107.8 \times 10^6 \text{ lb-in.}^3$$

The requirement for the impulsive load, expressed as M_pA, is adequately satisfied for both clamped and simply supported conditions. Also, the quasi-static shearing load of the beam ends is satisfied where V has been computed on the basis of uniform shear in the 10-in. webs (sides) of the beam with a small portion of the 8-in. section assumed to be effective in shear as well. Note that the plastic moment required for the quasi-static loading falls somewhere inbetween that required for clamped and simply supported beams. Even though the members in the roof have less bending rigidity than the side beams (as will be explained shortly, the 8 X 10-in. box beams without the reinforcing plates were used in the roof and floor), the end condition is more nearly clamped than simply supported, and, for the preliminary selection, the plastic moment developed by the reinforced box beam was judged to be

*This stress is slightly less than the minimum ultimate shearing stress (34,400 psi) computed as 60% of the minimum tensile strength of the material.

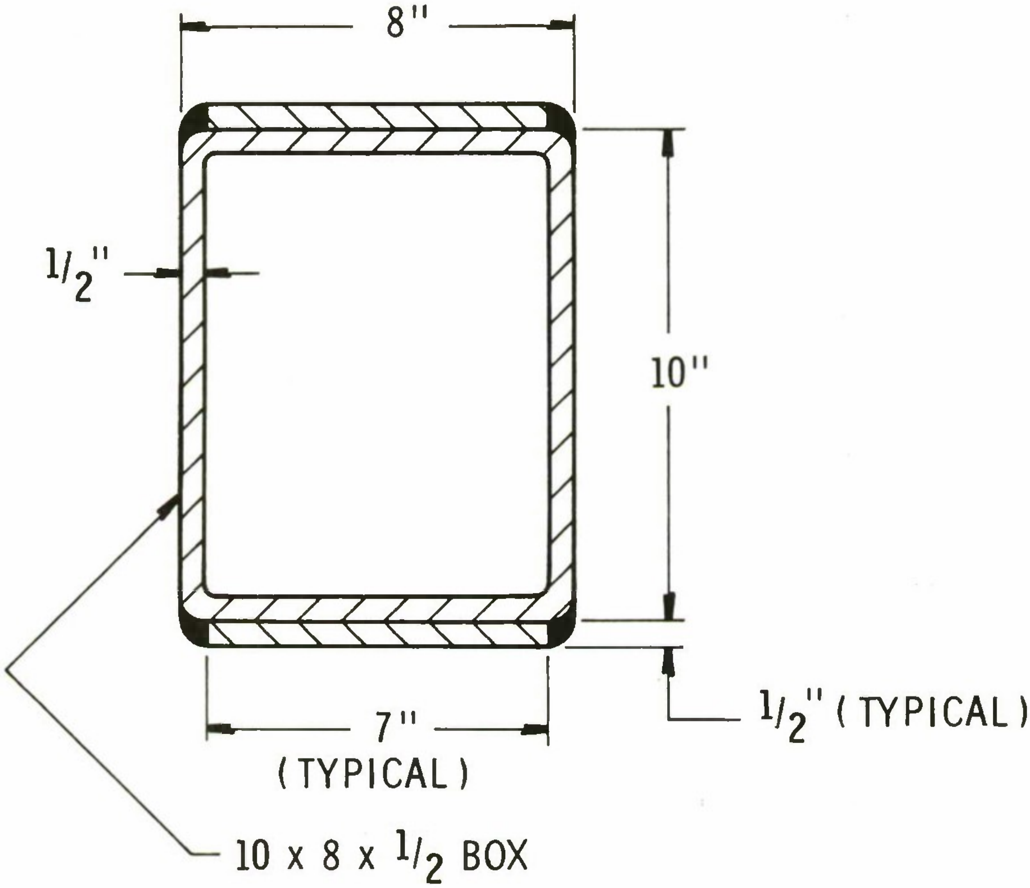


FIGURE 6. CROSS-SECTION OF FRAME VERTICAL MEMBERS

adequate. Tensile stresses in the side members were also checked and found to be small (approximately 8,000 psi). Because of the uncertainty in the end conditions for the beam, which can have a significant effect on the stresses, the effect of the axial load was not considered in the preliminary selection of the beam properties. It will, of course, be included in the more exact analysis.

The shearing stress in the beam under the impulsive loading has not yet been addressed. It was computed on the basis of the maximum curvature corresponding to the deflection which first produces an equivalent fully plastic bending moment in the beam. This curvature is set by the assumed deflection shape. Deep beams with high bending stiffness were eliminated by this criterion, but it was not a factor for the compact box section chosen. Equations (8), (9), (14) and (15) were used for these calculations.

The roof members were sized based on an elastic distribution of stresses in the grid of intersecting beams. From this we estimated that the bending strength of the members in the grid should be approximately 60% of that for the side members. This happens to be very close to the strength of the 8 X 10-in. box member without reinforcing plates, and these members were chosen for the roof and floor. The non-vented floor is closed by a solid plate of 0.375-in. thickness designed to resist the loads in membrane action in the same manner as for the vented panels.

The frame attachments for the membrane plate panels are such that the inplane loads are transferred continuously through the panels to the corner members of the frame. Details of the panel-to-panel and panel-to-corner attachments are shown in Figure 7.

VI. DISCUSSION

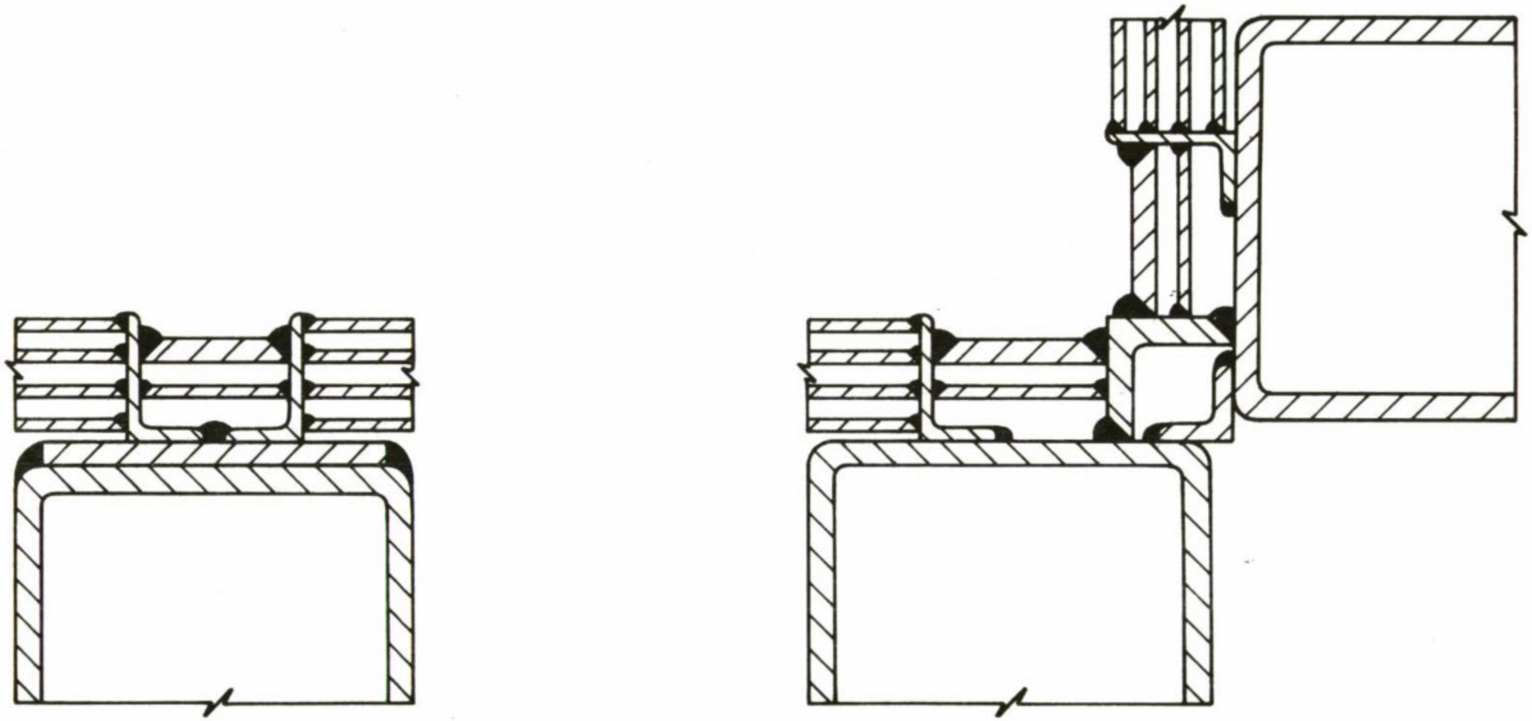
In this report, we have presented the status of the design of a suppressive structure which will first be tested in quarter-scale as part of a program to build and test a prototype Category I suppressive shield. This current quarter-scale frame design will be scrutinized further before it is finalized and tested. The work that is in progress* towards this end is:

(1) Possible panel redesign to fit the panels between the frame members. The panels were initially designed for other earlier frame concepts, and it is possible that modifications can be made to reduce panel size and simplify field installation.

(2) Along this same line, the frame may be slightly modified to simplify fabrication of the roof and floor. Figure 8 shows a variation of the frame in which the beams in the roof and floor would overlap instead of intersect.

(3) Additional analyses will be performed on the frame as soon as design modifications are finalized. Deformations and stresses will be computed for the governing quasi-static loading at maximum charge weight. This will be a static analysis in-

*Recall that the work reported here was done from January to September of 1974.



a. mid-wall connection

b. corner connection

FIGURE 7. TYPICAL PANEL INSTALLATION

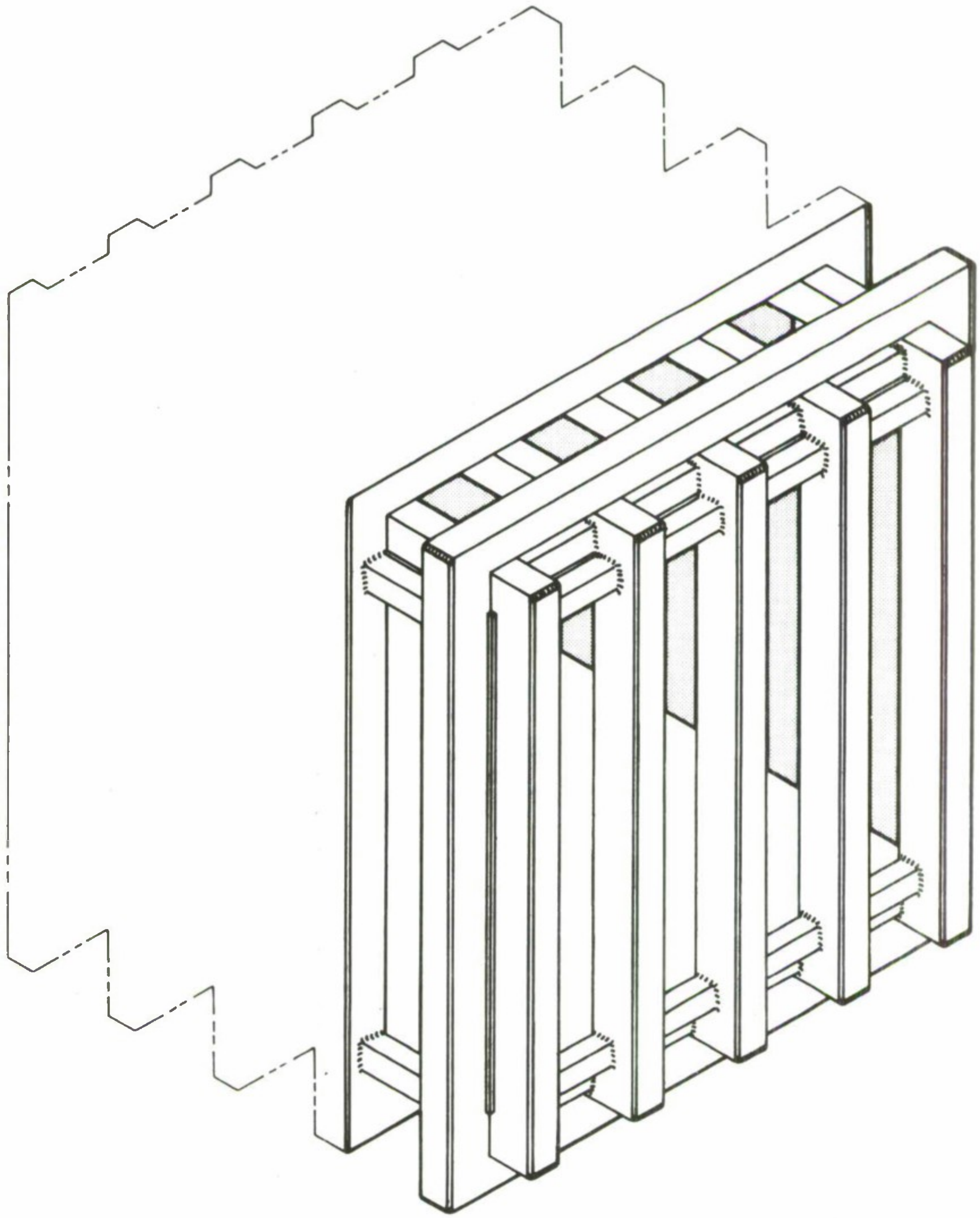


FIGURE 8. MODIFIED QUARTER SCALE FRAME

cluding large deflections and plastic strains. Elastic-plastic dynamic response for the impulsive loading will perhaps be determined also.

The panel and frame designs presented are just one phase in the development of the prototype structure. By conducting properly instrumented tests with the quarter-scale model we hope to improve the design of the prototype. Also, a long range Applied Technology Program being conducted by the Edgewood Arsenal should help fill in gaps in our knowledge concerning the fragment and blast threats, venting characteristics, inter-panel pressures, etc., so that future suppressive structures can in fact be routinely designed and applied to explosive processing operations.

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